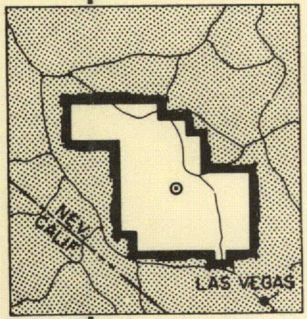
WT-1474

AEC Category: HEALTH AND SAFETY
Military Category: 32

OPERATION PLUMBBOB



NEVADA TEST SITE
MAY-OCTOBER 1957

Project 34.3

TEST OF BURIED STRUCTURAL-PLATE PIPES SUBJECTED TO BLAST LOADING

Issuance Date: July 28, 1961

CIVIL EFFECTS TEST GROUP

NOTICE

This report is published in the interest of providing information which may prove of value to the reader in his study of effects data derived principally from nuclear weapons tests.

This document is based on information available at the time of preparation which may have subsequently been expanded and re-evaluated. Also, in preparing this report for publication, some classified material may have been removed. Users are cautioned to avoid interpretations and conclusions based on unknown or incomplete data.

PRINTED IN USA
Price $1.25. Available from the Office of
Technical Services, Department of Commerce,
Washington 25, D. C.

USAEC Office of Technical Information Extension, Oak Ridge, Tennessee

Report to the Test Director

TEST OF BURIED STRUCTURAL-PLATE PIPES SUBJECTED TO BLAST LOADING

By

R. A. Williamson

and

P. H. Huff

Approved by:	L. J. VORTMAN Director Program 34	Approved by:	R. L. CORSBIE Director Civil Effects Test Group

Holmes & Narver, Inc.
Los Angeles, California
August 1960

ABSTRACT

Two 20-ft-long 7-ft-diameter 10-gauge structural-plate pipes, having longitudinal joints with eight bolts per foot, were buried and tested in the Smoky event of Operation Plumbbob at predicted pressure levels of 195 and 265 psi (actual pressure levels of approximately 190 and 245 psi). The depth of burial was 10 ft over the crown of the pipe.

Data obtained were to be used primarily in evaluating the suitability of structural-plate pipe as a partial substitute for heavy concrete tunnel sections in the construction of scientific stations at ranges close to nuclear detonations. However, in view of the nontypical soil conditions that exist at the Nevada Test Site, the results may not be applicable to other locations.

The principal measurements taken of each pipe were the transient changes in horizontal and vertical diameters vs. time, vertical and horizontal accelerations of the pipe invert, and the maximum interior overpressure. Supplementary data were the preshot and postshot measurements of horizontal and vertical diameters, joint slip, distances between end bulkheads, and elevations of the ground surface. Soil properties obtained included density, percentage of compaction, gradation, and the coefficient of internal friction.

Maximum transient changes in vertical and horizontal diameters, which were measured by self-recording gauges, were about $^7/_8$ in. and $^3/_8$ in., respectively. Maximum residual changes in the same diameters were about $^3/_4$ in. and $^1/_4$ in., respectively. Discrepancies were found between measurements of the residual changes recorded by the gauges and measurements obtained visually before and after the event. Slip in the bolted joints was negligible.

ACKNOWLEDGMENTS

The authors are indebted to many individuals and organizations for cooperation and assistance in the planning and execution of this project. The contributions and participation of the following individuals merit special commendation:

R. H. Carlson, Project Officer, Project 34.2
R. H. F. Boothe, Assistant Project Officer, Project 34.3
R. A. Cameron, Project Representative, Project 34.3
E. Y. Wong, Project Representative, Project 34.3
John Keefer, Ballistic Research Laboratories
W. L. Wood, Sandia Corporation
K. L. Cahill, Sandia Corporation

CONTENTS

CONTENTS (Continued)

ILLUSTRATIONS

ILLUSTRATIONS (Continued)

TABLES

Chapter 1

INTRODUCTION

1.1 OBJECTIVES

Project 34.3 was established to develop information of value in the structural design of test facilities for the U. S. Atomic Energy Commission at the Nevada Test Site (NTS) and the Eniwetok Proving Ground (EPG). The general purpose of the test was to determine the resistance of buried structural-plate pipe to high surface overpressures resulting from air bursts. This information was to be used primarily to evaluate the suitability of structural-plate pipe as a partial substitute for heavy concrete tunnel sections in the construction of scientific stations located at ranges close to nuclear detonations.

1.2 BACKGROUND

1.2.1 General

Structural-plate pipe is constructed of curved sectional corrugated-metal plates having longitudinal and circumferential bolted seams. Corrugations are nominally 2 in. deep and spaced 6 in. apart. The trade name Multiplate used by the Armco Steel Corporation to describe this pipe now has widely accepted usage, and it is used throughout this report synonymously with the term "structural plate."

1.2.2 Use of Multiplate Pipe in Tunnel Sections

Part of the work accomplished in connection with the scientific station program of Operation Plumbbob consisted in the design of buried reinforced-concrete tunnel sections intended to resist the close-in effects of air bursts (Plumbbob Station 9-22-6001 provides an example). In an attempt to reduce costs, Multiplate pipe was considered as a substitute for reinforced concrete in certain portions of the tunnels because preliminary estimates showed the possibility of worth-while savings in this application. Since protection of human life would not be a design factor in this case, the criterion for pipe selection could be less stringent than that required in shelter design. Large deflections could be tolerated so long as minimum postshot accessibility requirements were maintained.

The blast resistance of Multiplate pipe was known to be much greater than that predicted from its resistance to static loads. However, the overpressures to which this product had been exposed in previous operations were too low to give a satisfactory indication of its behavior at the high pressure levels that would be encountered in its proposed use.

Unfortunately calculation of the blast resistance of buried Multiplate pipe is unreliable because no known method exists by which to determine, even approximately, the transient load on the pipe. For this reason the proposal for its use as a partial substitute for heavy-concrete tunnel sections was abandoned temporarily. Later, in the interest of obtaining data in the higher pressure regions, the tests described in this report were initiated.

1.2.3 Static Strength of Multiplate Pipe

Multiplate pipe is classed in the general category of flexible metal pipe. When it is supporting earth cover, Multiplate pipe is capable of deflections sufficiently large to cause a favorable redistribution of external soil pressure before it fails. This redistribution increases its ability to resist static loads and largely invalidates theoretical attempts to determine stresses or collapsing loads. By the use of tests, observation, and experience, G. E. Shafer* concluded that failure of flexible pipe under static loading is due to excessive deflection rather than rupture of the pipe walls. From accumulated data, he developed an empirical method for determining the deflection of buried flexible pipe.[1]

M. G. Spangler[2] later developed a somewhat more rational method of determining pipe deflection. His method involved the modulus of passive resistance of the side fills (a factor that, in practice, generally cannot be determined with any degree of certainty).

Under present conditions the selection of a flexible pipe to carry a given static load is usually based on empirical data rather than on theoretical considerations.

1.2.4 Dynamic Strength of Multiplate Pipe

Reliance on empirical data is necessary in determining the resistance of buried flexible pipe to overpressure acting on the ground surface.

The blast resistance of buried Multiplate pipe was established for relatively low overpressures (below about 60 psi) in several past operations at NTS[3,4] and in Operation Greenhouse.[5] In many cases the pipes served as entries to larger structures, and diameter and wall thickness varied from 6 to 8 ft and from 3 to 10 gauge, respectively.

A buried 8-ft-diameter 10-gauge Multiplate pipe tested in Operation Plumbbob resisted an overpressure of well over 100 psi with no apparent distress.[6] This pipe was under an earth cover of 7.5 ft; and its longitudinal seams contained four bolts per foot, which is standard for this gauge pipe. On the other hand, repeated blast loadings on a 7.5-ft-diameter 10-gauge Multiplate pipe at NTS (Station 7-91.6 of Operation Buster-Jangle, 1951) under an earth cover of about 3 ft caused considerable flattening of the top portion of the pipe.

A static analysis (see Appendix A) was used in roughly assessing the strength of the pipe selected for these tests. Analysis is based on the assumptions that a part of the overpressure is resisted by the earth cover and that the remainder of the load is transmitted to the pipe in a uniform radial pressure, causing failure of the bolted joints or crushing of the pipe-wall material. The calculations indicate that failure might occur at an overpressure of less than 200 psi.

REFERENCES

1. G. E. Shafer, Discussion of an article by M. G. Spangler entitled Underground Conduits, An Appraisal of Modern Research, Proc. Am. Soc. Civil Engrs., 74: 267 (February 1948).
2. M. G. Spangler, Underground Conduits, An Appraisal of Modern Research, Proc. Am. Soc. Civil Engrs., 73: 855-884 (June 1947).
3. R. B. Vaile, Jr., and L. D. Mills, Evaluation of Earth Cover as Protection to Aboveground Structures, Project 3.6, Bureau of Yards and Docks, Operation Teapot Report, WT-1128, December 1956 (Classified).
4. R. M. Longmire and L. D. Mills, Navy Structures, Projects 3.11 to 3.16, Bureau of Yards and Docks, Operation Upshot–Knothole Report, WT-729, May 1955 (Classified).
5. U. S. Army Structures Test, Operation Greenhouse Interim Report, Annex 3.1, Vols. I and II (Classified) and III, 1951.
6. G. H. Albright, Evaluation of Buried Conduits as Personnel Shelters, Project 3.2, Bureau of Yards and Docks, Operation Plumbbob Report, WT-1421, November 1957.

*Chief Engineer, Armco Drainage & Metal Products, Inc.

Chapter 2

PROCEDURE

2.1 SCOPE OF PROJECT

As finally approved the project consisted in the testing of two 20-ft lengths of pipe buried under 10 ft of earth cover. They were designated as Stations 8-34.3-8018.01 and 8-34.3-8018.02 in Area 8, and in this report they will be referred to as Stations 1 and 2. These stations, located at ranges of 825 and 900 ft, respectively, from the Ground Zero (GZ) of the Smoky event, were expected to be subjected to overpressures of 265 and 195 psi, respectively. Details of the stations are given in Fig. 2.1. Figure 2.2 shows an interior view of Station 1.

2.2 SOIL CONDITIONS

The unusual soil conditions indigenous to NTS were encountered at the location selected. Beneath a thin surface layer of soil was a hard stratum of conglomeritic gravel particles about 1 in. maximum in size bound into a matrix by a natural cementitious material several feet thick. Blasting was necessary to penetrate this stratum. Below it was a deposit of dense, fairly well cemented sand and gravel, which was excavated with a backhoe.

Material that met specification requirements for backfill was obtained from a site about 1.4 miles away. Machine tamping was used for compaction.

The soil characteristics necessitated the excavation of a minimum-sized hole in order to reduce costs and caused test conditions to be classed generally as nontypical.

2.3 PIPE

The pipe used was 7-ft-diameter 10-gauge galvanized Multiplate, which was furnished by Armco. The pipe material was copper-bearing iron, with a minimum tensile yield strength of 27 kip/sq in. Joint strength more nearly consistent with the strength of the pipe walls was provided by using eight bolts per foot in longitudinal seams instead of the usual four bolts per foot. This does not represent a radical departure from standard practice since eight bolts per foot are used with 1-gauge Multiplate and six bolts per foot are commonly used with intermediate gauges.

Torque wrenches were used to tighten the bolts, which were $^3/_4$-in.-diameter high-strength bolts. The specified torque was 320 lb-ft minimum.

2.4 END WALLS

The timber end walls, shown in Figs. 2.1 and 2.3, incorporated features that were intended to minimize end effects due to blast loading. One feature was the use of several thicknesses of

asphalt-impregnated fiberboard on the face against the end of the pipe to provide, as far as was feasible, free movement of the pipe wall.

A second feature consisted in extending the timbers several feet outside the pipe walls to engage a large amount of earth fill. The purpose of this was an attempt to resist dynamic earth pressure against the ends of the pipe without causing high axial stresses in the pipe.

2.5 INSTRUMENTATION

2.5.1 General

The instrumentation, which was furnished by Sandia Corporation, consisted of two self-recording deflection gauges[1,2] in each pipe to measure transient changes in horizontal and vertical diameters and a peak-pressure indicator for roughly measuring interior overpressures. The Ballistic Research Laboratories (BRL) later became interested in investigating the performance of a new type self-recording accelerometer; and, by mutual agreement, one of these instruments was installed at the invert of the forward station.

For triggering the self-recording instruments, a 2.5-sec timing signal was provided by Edgerton, Germeshausen, and Grier (EG&G). This firm also furnished a number of film badges for measuring interior radiation.

2.5.2 Self-recording Deflection Gauges

Deflection gauges were obtained from the military, modified by a special linkage system to record diameter changes up to 24 in., and mounted in the pipe (Fig. 2.2). The mounting arrangement consisted of an interior plate bolted through the corrugations to another plate outside the pipe. This arrangement was used at each end of the horizontal and vertical diameters (Fig. 2.1). The modified gauges were similar to those used in Project 34.2 of Operation Plumbob.[3] The natural frequency of these gauges was approximately 17 cycles/sec.

2.5.3 Peak-pressure Indicators

The peak-pressure indicators, constructed at Sandia Corporation, consisted of paper membranes stretched over holes of varying diameter. Rupture of the membrane over a given hole takes place at an experimentally predetermined overpressure. Figures 2.4 and 2.5 show one of these gauges. The largest of these gauges was 14 in. in diameter and was designed to rupture at $\frac{1}{2}$ psi.

2.5.4 Accelerometer

The experimental accelerometer (Fig. 2.6), installed by BRL in Station 1, inscribed a record of horizontal and vertical acceleration on the silvered surface of a rotating disk actuated by a battery-powered d-c motor. The accelerometer was activated by the 2.5-sec EG&G timing signal.

2.5.5 Film Badges

Several film badges, which were used to determine interior radiation levels, were provided in both stations by EG&G. They were suspended from the crown of the pipe (Fig. 2.2).

2.6 PRESHOT AND POSTSHOT MEASUREMENTS

Preshot and postshot measurements of the diameters and lengths of the pipes, joint slip, and elevations of the ground surface are presented in the tables and figures in Chap. 3.

2.6.1 Diameter Measurements

Inside horizontal and vertical diameters at the middle of the pipe were obtained to the nearest 0.001 ft between reference marks on the pipe shell. A level rod was adapted for this purpose by inserting a screw into the metal shoe at each end; this arrangement provided, in effect, a large inside caliper.

Readings were taken before backfilling with the fill level with the top of the pipe and finally with the fill completed. This particular sequence was followed to obtain data for estimating the value of the passive modulus of the side fill.

The third measurement was repeated before the event to check previous readings and to determine any lag or delayed loading, which sometimes causes creep of flexible pipe. The lengths of the two slant diameters, inclined at approximately 45° to the horizontal at this same section, also were obtained both before and after the event.

2.6.2 Joint-slip Measurements

Slip in all bolted longitudinal seams was measured from a series of scratch marks (at each longitudinal seam) located in the vertical plane of the circumferential seams. The amount of slip was obtained by determining the number of marks covered by the movement of the joint.

Slip in the bolted circumferential seams was measured by a similar method.

2.6.3 Distances Between End Bulkheads

The distances between end bulkheads before and after the event were measured with a steel tape along element lines at the ends of the vertical and horizontal diameters.

2.6.4 Elevations of Ground Surface

The elevations of five buried monuments located about 12 in. below the ground surface were obtained before and after the event at each station.

These monuments were located in a vertical plane normal to the axis of the pipe and midway between the ends. One monument was placed directly above the axis of the pipe; the remaining four were located symmetrically with respect to the first. Two were located just within the limits of the excavation, and two were located just beyond the limits of the excavation. The bench mark used was situated about 5200 ft away from Station 1 at a range of about 6000 ft from GZ. At this range it was believed that any settlement of the bench mark would be a small percentage of the monument settlements.

2.6.5 Invert Elevations

Preshot elevations of the pipe invert were recorded. However, the elevations were obtained relative to a point whose preshot absolute elevation was not recorded; thus the possibility of obtaining usable postshot elevations was prevented.

2.7 SOIL PROPERTIES

The properties of the backfill material were determined through field and laboratory tests by the International Testing Corporation of Long Beach, under contract to the Smith-Emery Co. of Los Angeles. These properties included unit weight, percentage of compaction, angle of internal friction, and gradation.

The percentage of compaction was obtained at levels both below and above the top of the pipe.

REFERENCES

1. P. A. Northrop, Instrumentation for Structures Program, Part I, Annex 3.4, Sandia Corporation, Operation Greenhouse Report, WT-1, January 1951.
2. P. A. Northrop, Instrumentation for Structures Program, Part II, Annex 3.4, Sandia Corporation, Operation Greenhouse Report, WT-10, August 1951 (Classified).
3. R. H. Carlson and J. P. Murtha, Comparison Test of Reinforcing Steels, Project 34.2, Operation Plumbbob Report, WT-1473, September 1960.

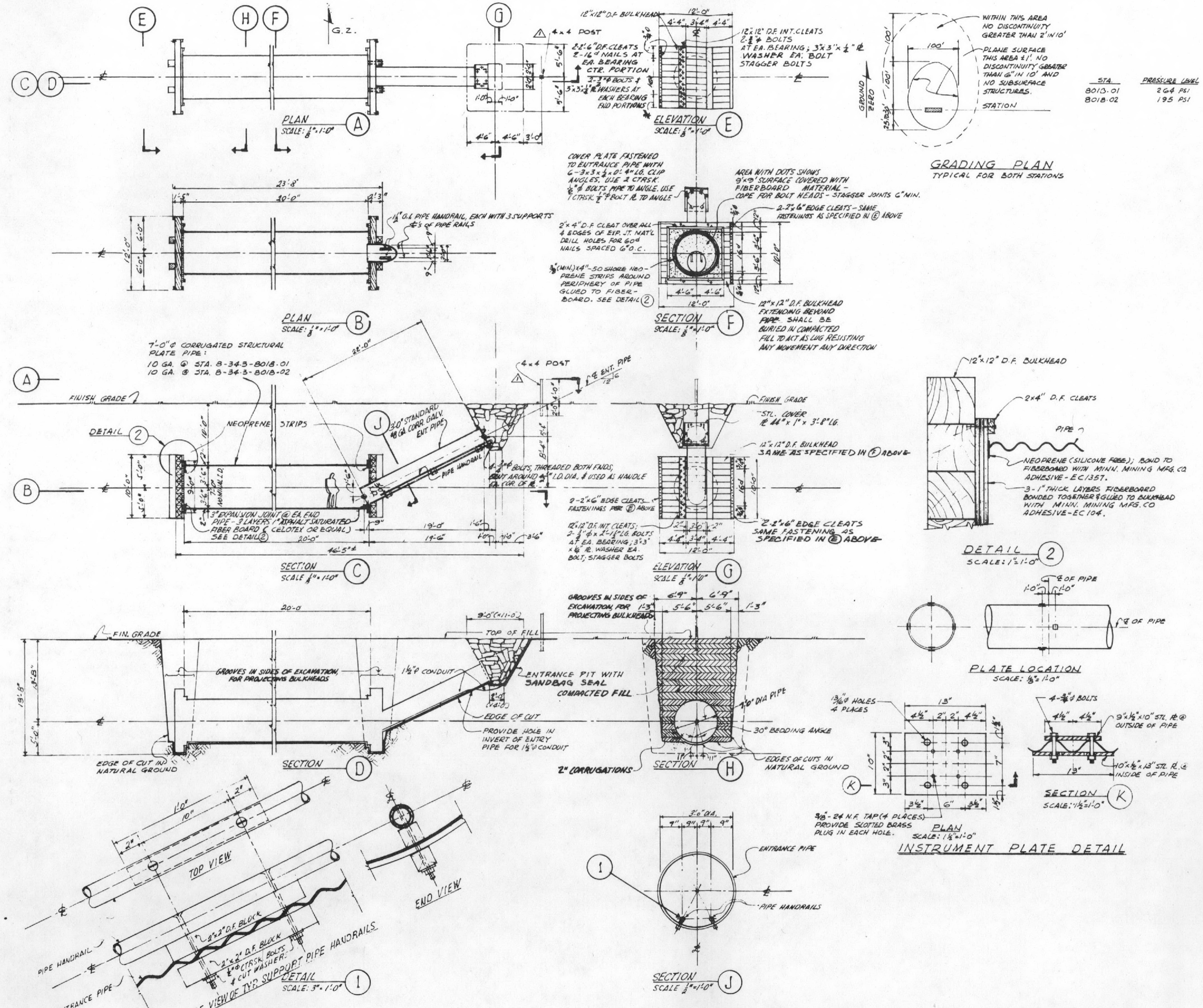


Fig. 2.1 — Project 34.3 buried-pipe tests: plans, sections, and details.

PIPE MATERIALS

1. Pipe shall be furnished in conformance with requirements of Division 4, Section 19, of the standard specification for highway bridges of AASHO, 6th ed., and the additional requirements here specified.

2. Minimum guaranteed tensile yield strength and ultimate tensile strength of the pipe material shall be specified by the manufacturer.

3. Bolts shall be high strength bolts of not less than 3/4" diameter. Bolt and nut materials shall conform to the requirements of the current edition of ASTM A-325. Calibrated power or torque wrenches shall be used for tightening bolts. Bolts shall be tightened to a tension of not less than 29,000 lb or torque of not less than 320 ft-lb. Shaped bolt heads and nuts shall be used, or standard type bolts and nuts with shaped washers shall be used in lieu thereof.

4. Longitudinal seams shall have not less than 8 bolts per foot.

WOOD

1. Lumber shall be 1500F Douglas fir grade marked, conforming to WCLA grading rules #15.

STRUCTURAL STEEL

1. All structural steel shall conform to the current edition of ASTM A-7.

PLACEMENT OF PIPE

1. The pipe shall be installed so that longitudinal seams are located symmetrically with respect to the vertical diameter of the pipe. Longitudinal seams shall not occur at the horizontal or vertical diameters in the midlength portion of the pipe.

EXCAVATION AND BACKFILL

1. Backfill material shall be free from large rocks and large lumps or clods larger than 3" in diameter. Backfill material used shall be subject to the approval of the project representative before placement. Fill shall be compacted to 90% optimum density, as determined in accordance with AASHO Standard Method T-99.

2. Fill material shall be compacted in 6" layers and placed alternately on both sides of pipe. Fill above top of pipe shall not be placed until after field measurements are made by Holmes & Narver field forces.

3. If feasible, fill material against each face of wood bulkheads shall be placed and compacted so as to prevent undue penetration of the fiberboard material by the end of the pipe.

REQUIRED SOIL TESTS

1. The following data shall be obtained relative to the compacted fill material below the level of the top of the pipe at both stations prior to the event: (1) percentage of compaction, (2) unit weight, (3) coefficient of internal friction, and (4) soil classification. Results shall be presented in report form.

2. The same information is required for the fill material above the top of the pipe. After the event, the percentage of compaction and unit weight of the fill material above the top of the pipe shall be rechecked.

FIELD MEASUREMENTS

1. The following data shall be obtained by Holmes & Narver field forces at both stations. Measurements of inside vertical and horizontal diameters at the midlength of the pipe: (1) prior to backfilling, (2) with backfill at the top of pipe, (3) with backfill completed, (4) within a few days of the event, and (5) after the event. Measurements shall be taken to the nearest 0.001 ft between clearly marked points on the corrugations.

2. Invert elevations to the nearest 0.01 ft, at ends and midlength of the pipe, obtained when diameters are measured.

3. Average width of excavation in the middle third of the pipe length measured at the top of the pipe.

4. Elevation of buried monuments located in a vertical plane at the midlength of the pipe, normal to the pipe axis. Monuments shall be located about 1 ft below finished grade. One directly over the pipe axis and two on each side of the pipe axis, one just within and one just beyond the limits of the excavation. Elevations shall be recorded before and after the event.

CONSTRUCTION PHOTOGRAPHS

1. Holmes & Narver field forces shall furnish construction photographs when: (1) excavation is completed and pipe is in place, (2) end bulkheads and entrance pipe are installed, and (3) fill is completed and before sandbags are placed.

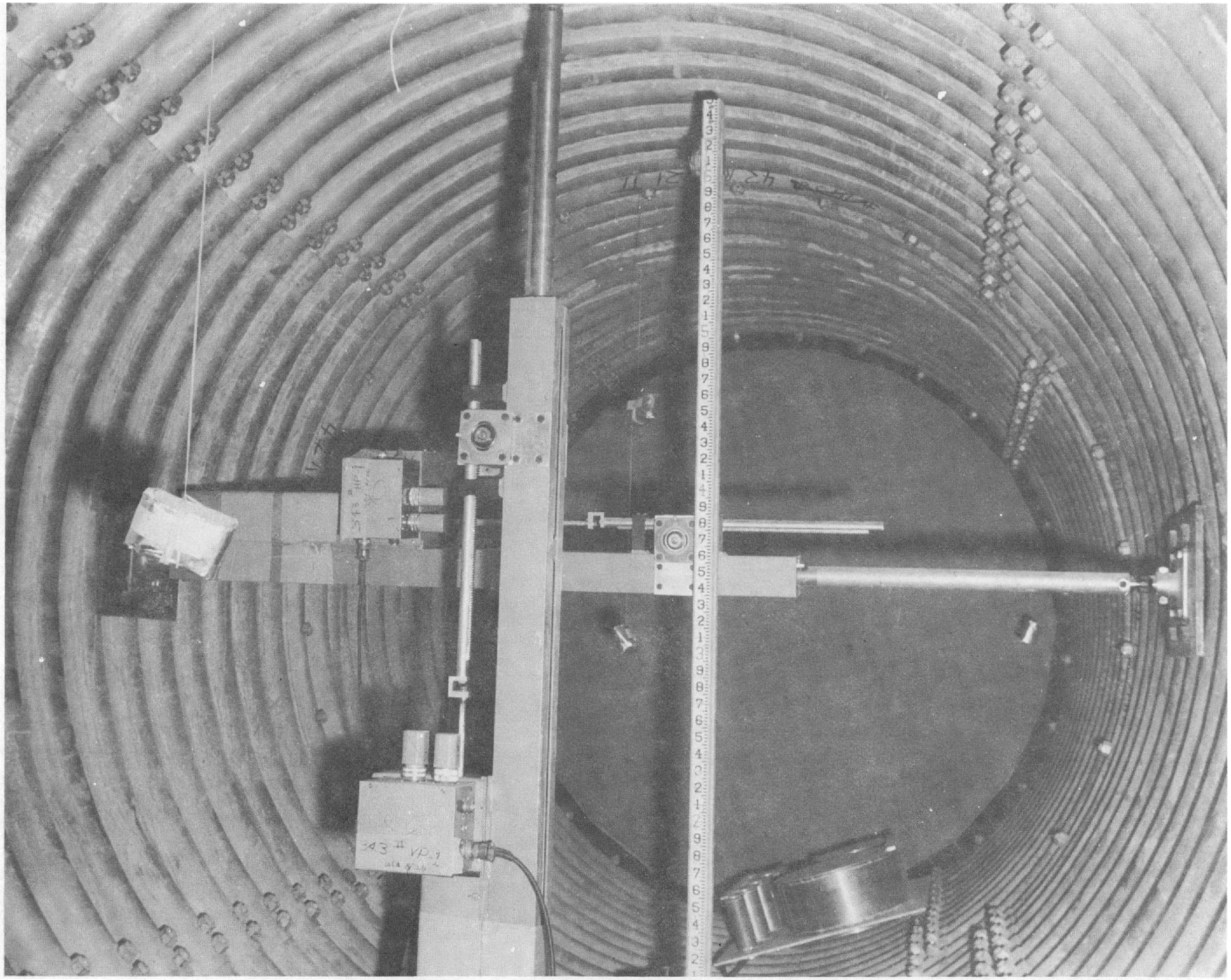


Fig. 2.2—Deflection gauges and film badges installed in pipe.

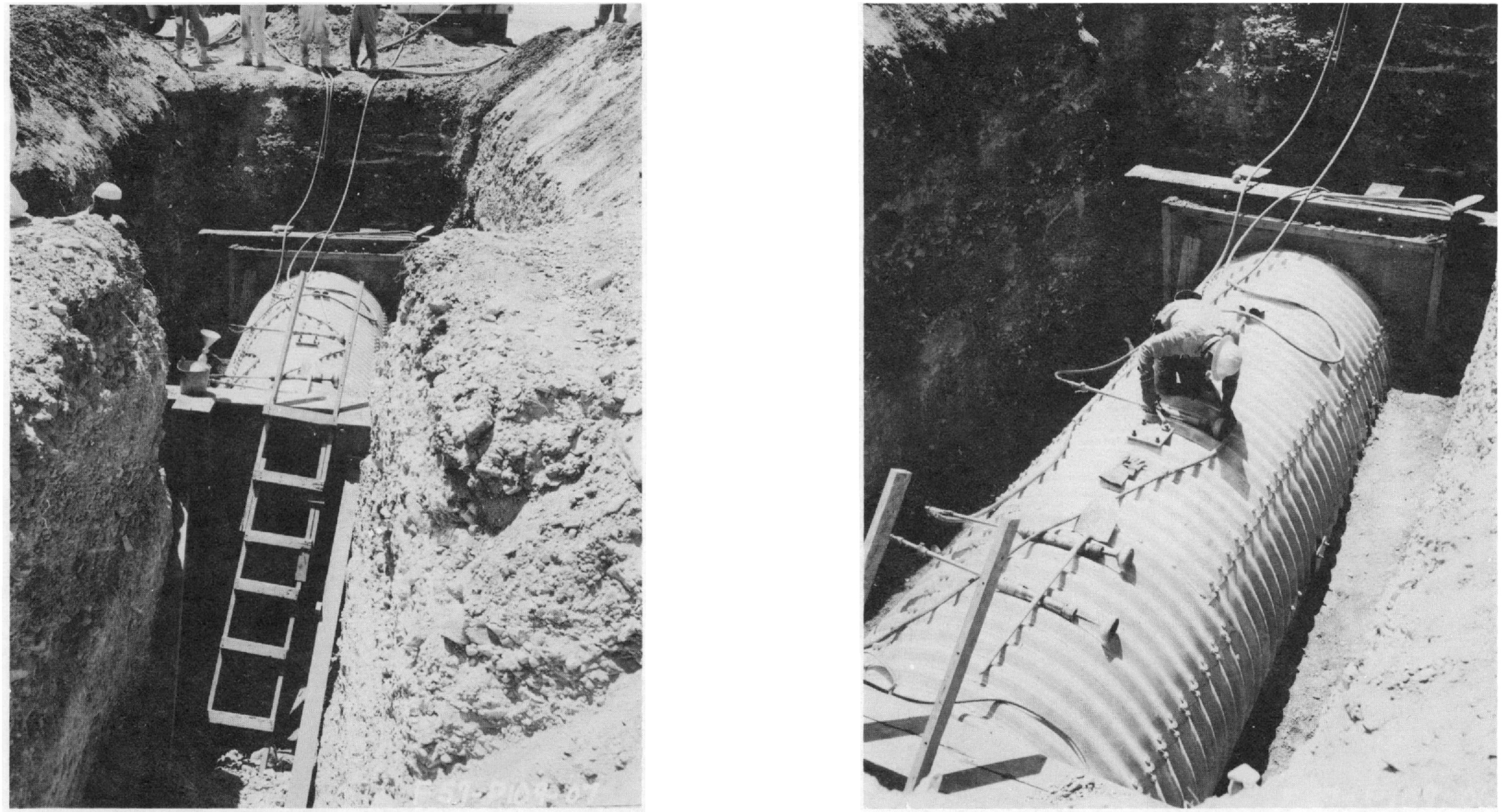


Fig. 2.3—Exterior views of pipe in place prior to placing backfill.

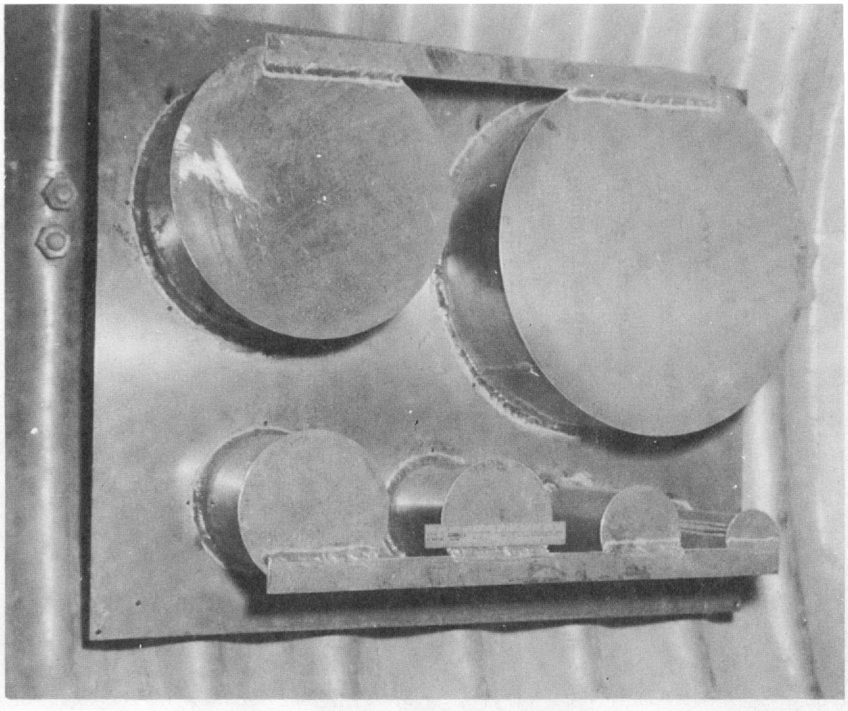


Fig. 2.4 — Peak-pressure indicator, top view.

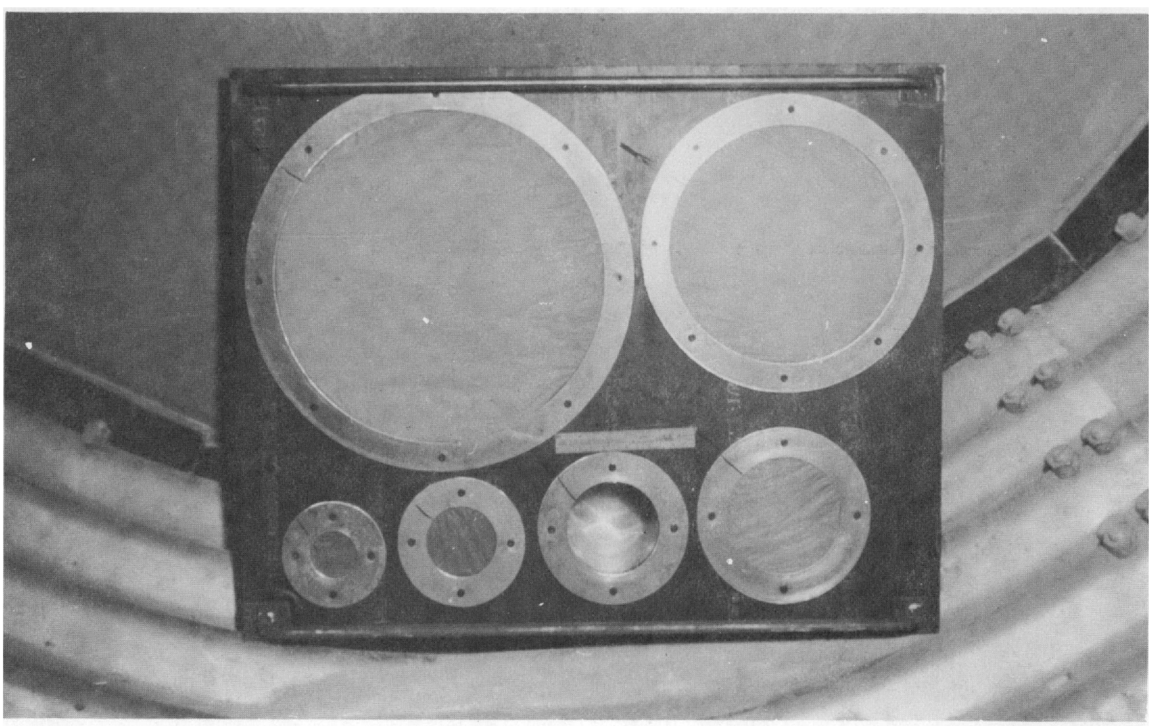


Fig. 2.5 — Peak-pressure indicator, underside.

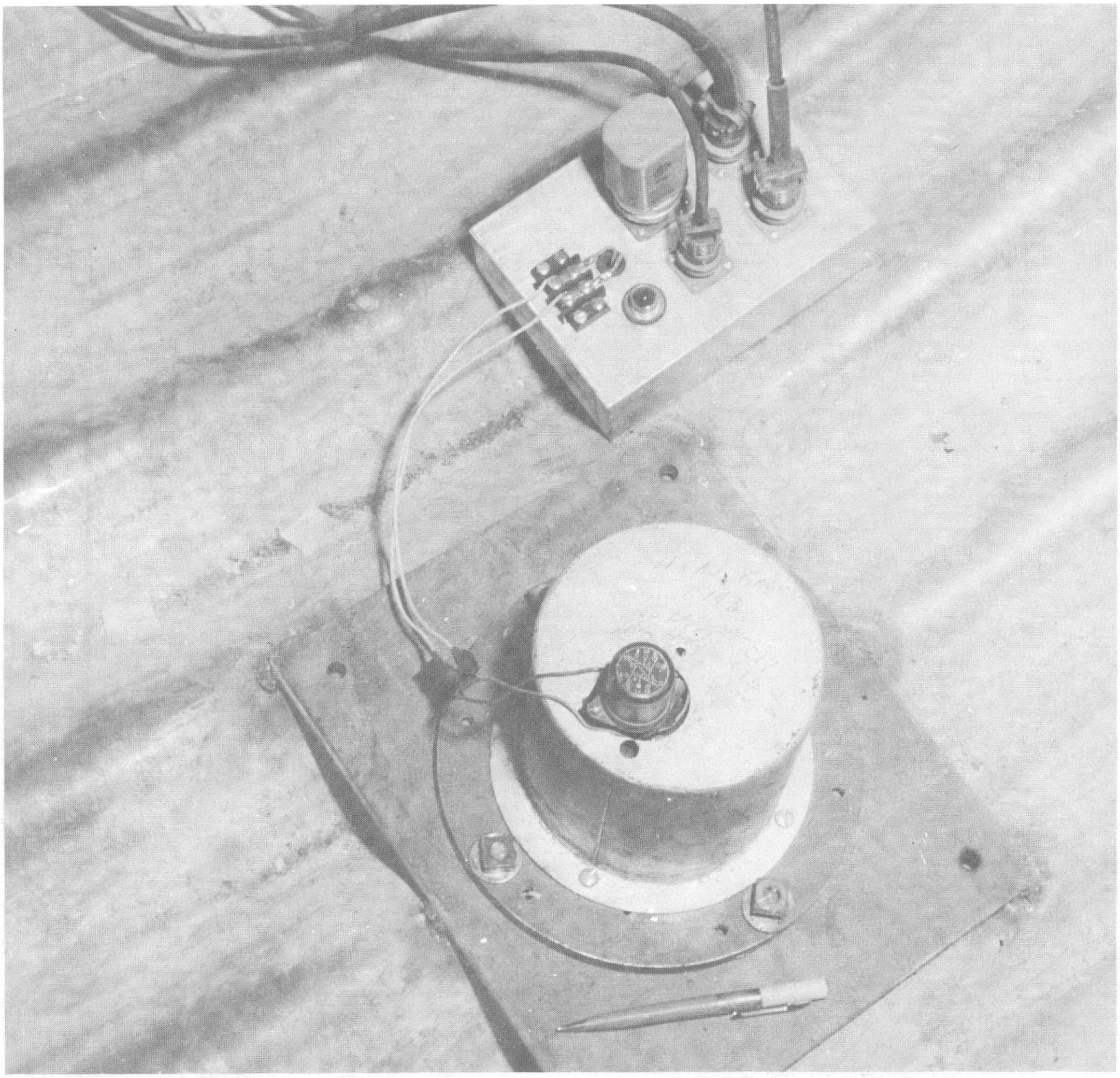


Fig. 2.6—Accelerometer mounted in Station 1.

Chapter 3

RESULTS

3.1 OVERPRESSURES

No surface pressure-time instrumentation was included in this project. However, a blast line in the area was instrumented as part of Project 1.8a, and overpressure vs. time data were extrapolated from those records (as shown in Fig. 3.1). On this basis, 245 and 190 psi were the maximum surface overpressures at Stations 1 and 2, respectively.

None of the interior peak-overpressure diaphragms ruptured, indicating that the maximum interior overpressure at both stations was less than 0.5 psi.

3.2 TRANSIENT DIAMETER CHANGES

The significant portion of the record traced by the self-recording deflection gauges is reproduced in Figs. 3.2 and 3.3. Peak values and residuals are summarized in Table 3.1.

3.3 INVERT ELEVATIONS

As noted in Sec. 2.6.5, no usable postshot elevations were obtained because the absolute preshot elevation of the reference point was not determined.

3.4 ACCELEROMETER RECORDS

The trace of the accelerometer record is shown in Fig. 3.4. Only the initial 100 msec of the vertical accelerometer record was considered valid because spring hysteresis caused a base-line shift throughout the latter portion of the record. Only the first 58 msec of the horizontal acceleration record was obtained.

3.5 COMPUTED INVERT VELOCITIES AND DISPLACEMENTS

Horizontal and vertical velocities of the pipe invert were obtained by integration of the acceleration vs. time records, and integration of the resulting velocity-time curves gave a plot of the displacement of the invert. These curves are plotted in Figs. 3.5 and 3.6.

3.6 OTHER MEASUREMENTS

Table 3.2 records the preshot and postshot measurements of horizontal and vertical diameters. Corresponding measurements of the slant diameters are presented in Table 3.3. The locations of monuments and the preshot and postshot elevations of the monuments are

shown in Table 3.4. Table 3.5 lists the preshot and postshot distances between end bulkheads. Table 3.6 is a summary of the most significant data obtained.

Figure 3.7 shows interior radiation dosage accumulated in the $1\frac{1}{2}$-month period between the time of the event and the time of the recovery of the film badges.

3.7 SOIL PROPERTIES

The backfill material was sandy gravel with a maximum density of 122 lb/cu ft at a moisture content of 10.5 per cent. The average field density was about 114 lb/cu ft, and the average compaction attained was about 93 per cent of maximum.

Direct shear tests indicated an internal coefficient of friction of 1.42 in one case and 0.875 in another.

These results are reported in greater detail in Appendix B.

TABLE 3.1 — PEAK TRANSIENT AND RESIDUAL DIAMETER CHANGES FROM SELF-RECORDING GAUGES

Location	Vertical*	Horizontal*
	Station 1	
Peak:		
Feet	−0.074	+0.032
Inches	−0.89	+0.39
Residual:		
Feet	−0.057	+0.019
Inches	−0.68	+0.23
	Station 2	
Peak:		
Feet	−0.028	+0.035
Inches	−0.34	+0.41
Residual:		
Feet	−0.0025	+0.016
Inches	−0.03	+0.19

*Increases are indicated by +; decreases, by −.

TABLE 3.2—VERTICAL AND HORIZONTAL DIAMETERS, PRESHOT AND POSTSHOT VISUAL MEASUREMENTS

Condition	Inside diameter, ft: Vertical	Inside diameter, ft: Horizontal	Successive change in diameter, ft*: Vertical	Successive change in diameter, ft*: Horizontal
		Station 1		
Preshot, no fill in place	6.916	6.960		
			+0.003	−0.008
Preshot, backfilled to top of pipe	6.919	6.952		
			−0.001	+0.003
Preshot, backfill complete: (8-1-57)	6.918	6.955		
			+0.001	−0.001
(8-12-57)	6.919	6.954		
			−0.015	+0.005
Postshot (10-16-57)	6.904	6.959		
		Station 2		
Preshot, no fill in place	6.945	6.812		
			+0.010	−0.003
Preshot, backfilled to top of pipe	6.955	6.809		
			−0.002	+0.005
Preshot, backfill complete: (8-4-57)	6.953	6.814		
			0.000	+0.001
(8-12-57)	6.953	6.815		
			−0.013	+0.04
Postshot (10-17-57)	6.940	6.855		

*Increases are denoted by +; decreases, by −.

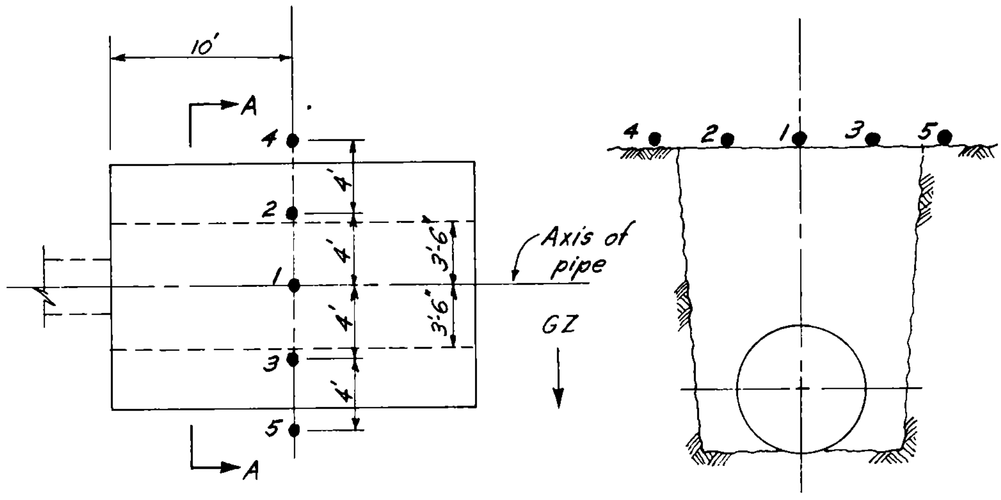
TABLE 3.3—SLANT DIAMETERS, PRESHOT AND POSTSHOT VISUAL MEASUREMENTS

Condition	Inside diameter,* ft: A	Inside diameter,* ft: B	Change in diameter,† ft: A	Change in diameter,† ft: B
		Station 1		
Preshot	6.735	6.773		
			−0.021	+0.016
Postshot (10-16-57)	6.714	6.789		
		Station 2		
Preshot	6.813	6.756		
			+0.024	−0.062
Postshot (10-17-57)	6.837	6.694		

*Slant diameter A is inclined nominally at 45° to the horizontal with the lower end nearer GZ. Diameter B is nominally at 90° to diameter A. Measurements are taken between marks on projecting bolt heads.

†Increases are indicated by +; decreases, by −.

TABLE 3.4—PRESHOT AND POSTSHOT MONUMENT ELEVATIONS



PLAN SECTION A-A

	Preshot elevation, ft	Postshot elevation, ft	Net changes, ft
		Station 1	
Monument 1	44.36	44.09	−0.27
2	44.26	44.04	−0.22
3	44.74	44.50	−0.24
4	44.21	44.10	−0.11
5	44.76	44.64	−0.12
		Station 2	
Monument 1	41.91	41.62	−0.29
2	41.69	41.44	−0.25
3	41.86	41.58	−0.28
4	41.60	41.51	−0.09
5	42.00	41.88	−0.12

TABLE 3.5—DISTANCES BETWEEN END BULKHEADS

	Vertical diameter		Horizontal diameter		Average
Condition	Top	Bottom	Front	Rear	distance
			Station 1		
Preshot	20 ft 2$\frac{5}{8}$ in.	20 ft 2$\frac{5}{8}$ in.	20 ft 2$\frac{7}{8}$ in.	20 ft 2$\frac{5}{8}$ in.	20 ft 2$\frac{11}{16}$ in.
Postshot	20 ft 2$\frac{3}{8}$ in.	20 ft 2$\frac{3}{8}$ in.	20 ft 2$\frac{5}{8}$ in.	20 ft 2$\frac{5}{16}$ in.	20 ft 2$\frac{7}{16}$ in.
Change	−$\frac{1}{4}$ in.	−$\frac{1}{4}$ in.	−$\frac{1}{4}$ in.	−$\frac{5}{16}$ in.	
			Station 2		
Preshot	20 ft 2$\frac{3}{4}$ in.	20 ft 2$\frac{7}{8}$ in.	20 ft 2$\frac{3}{4}$ in.	20 ft 3 in.	20 ft 2$\frac{7}{8}$ in.
Postshot	20 ft 4$\frac{1}{2}$ in.	20 ft 3$\frac{1}{16}$ in.	20 ft 2$\frac{13}{16}$ in.	20 ft 3$\frac{1}{2}$ in.	20 ft 3$\frac{7}{16}$ in.
Change	+1$\frac{3}{4}$ in.	+$\frac{3}{16}$ in.	+$\frac{1}{16}$ in.	+$\frac{1}{2}$ in.	

TABLE 3.6 — SUMMARY OF PRINCIPAL DATA

	Vertical and horizontal diameters*			
	Station 1		Station 2	
	Vertical	Horizontal	Vertical	Horizontal
Preshot diameter, ft	6.919	6.954	6.953	6.815
Postshot diameter, ft	6.904	6.959	6.940	6.855
Residual change, ft	−0.015	+0.005	−0.013	+0.040
Residual change from gauge, ft	−0.057	+0.019	−0.0025	+0.016
Discrepancy:				
Feet	+0.042	−0.014	−0.0105	+0.024
Inches	+0.50	−0.17	−0.12	+0.29
Transient change from gauge:				
Feet	−0.074	+0.032	−0.028	+0.035
Inches	−0.89	+0.39	−0.34	+0.41

	Slant diameters*			
	Station 1		Station 2	
	A	B	A	B
Preshot diameter, ft	6.735	6.773	6.813	6.756
Postshot diameter, ft	6.714	6.789	6.837	6.694
Residual change:				
Feet	−0.021	+0.016	+0.024	−0.062
Inches	−0.25	+0.19	+0.29	−0.74

	Average distance between end bulkheads	
	Station 1	Station 2
Preshot	20 ft $2\frac{11}{16}$ in.	20 ft $2\frac{7}{8}$ in.
Postshot	20 ft $2\frac{7}{16}$ in.	20 ft $3\frac{7}{16}$ in.
Change	$-\frac{1}{4}$ in.	$+\frac{9}{16}$ in.

	Joint slip	
	Station 1	Station 2
Longitudinal	None	None
Circumferential	None	None

*See footnotes, Table 3.3.

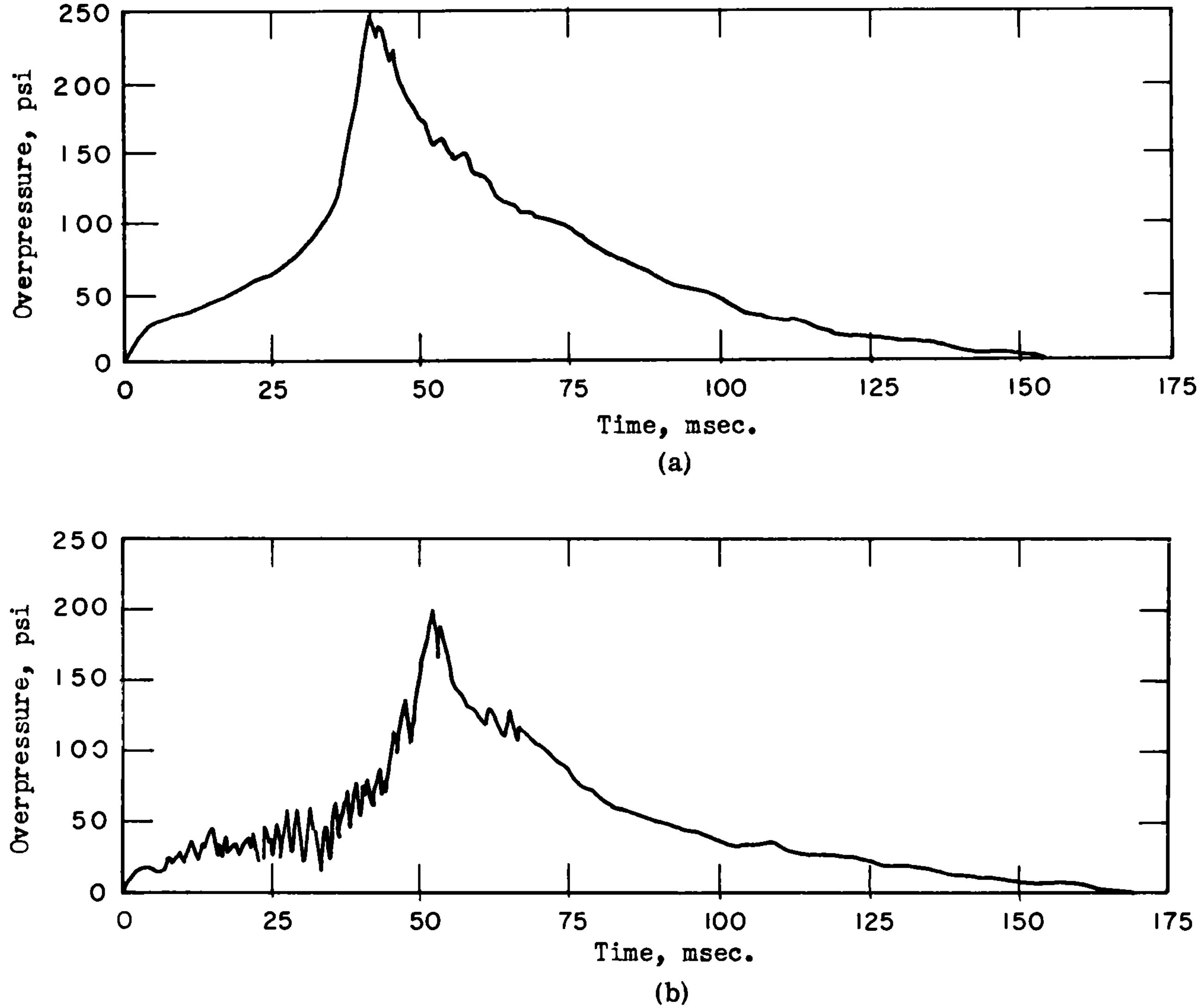


Fig. 3.1—Surface overpressure-time records. (a) At 825 ft. (b) At 900 ft.

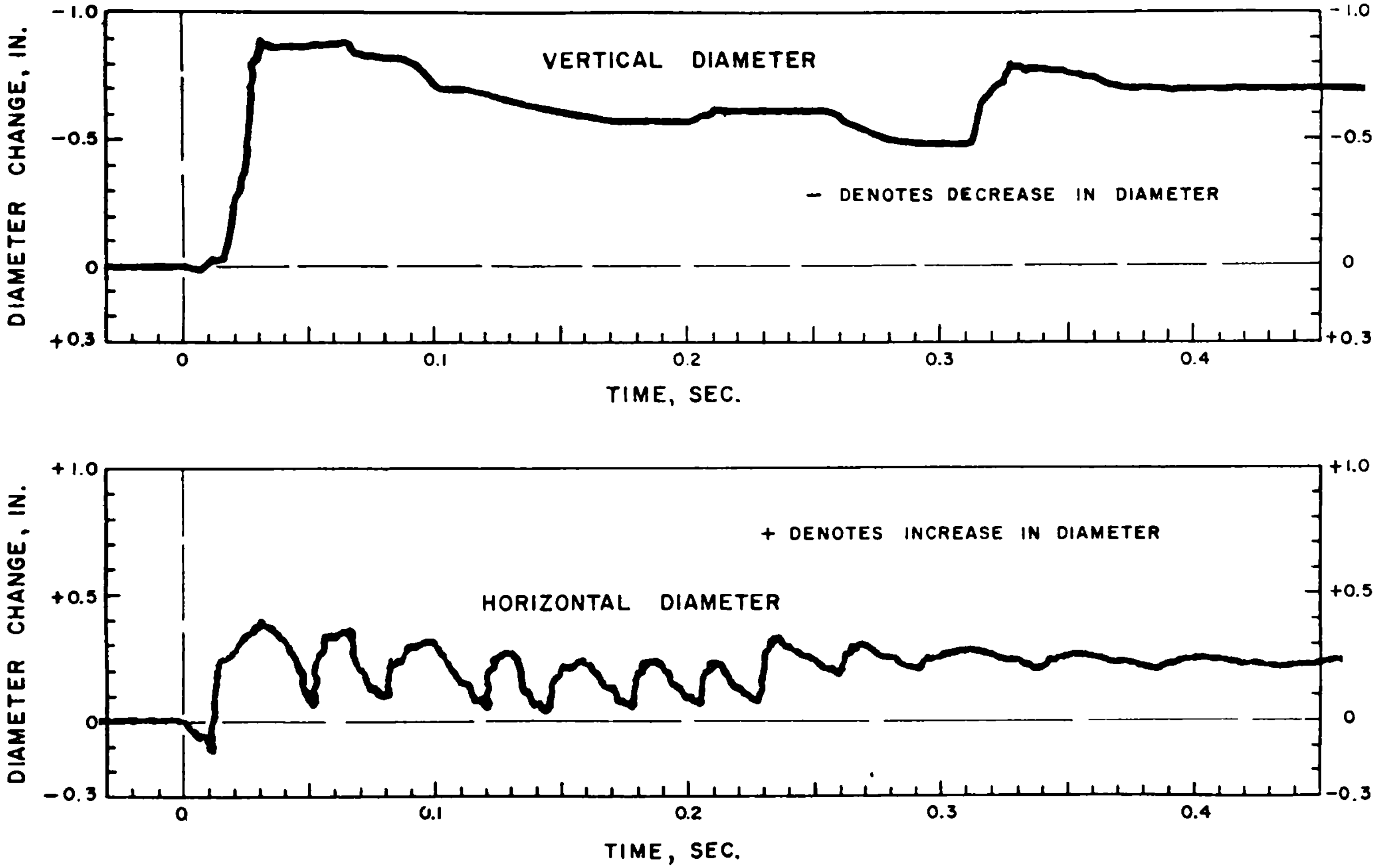


Fig. 3.2—Transient diameter changes, Station 1.

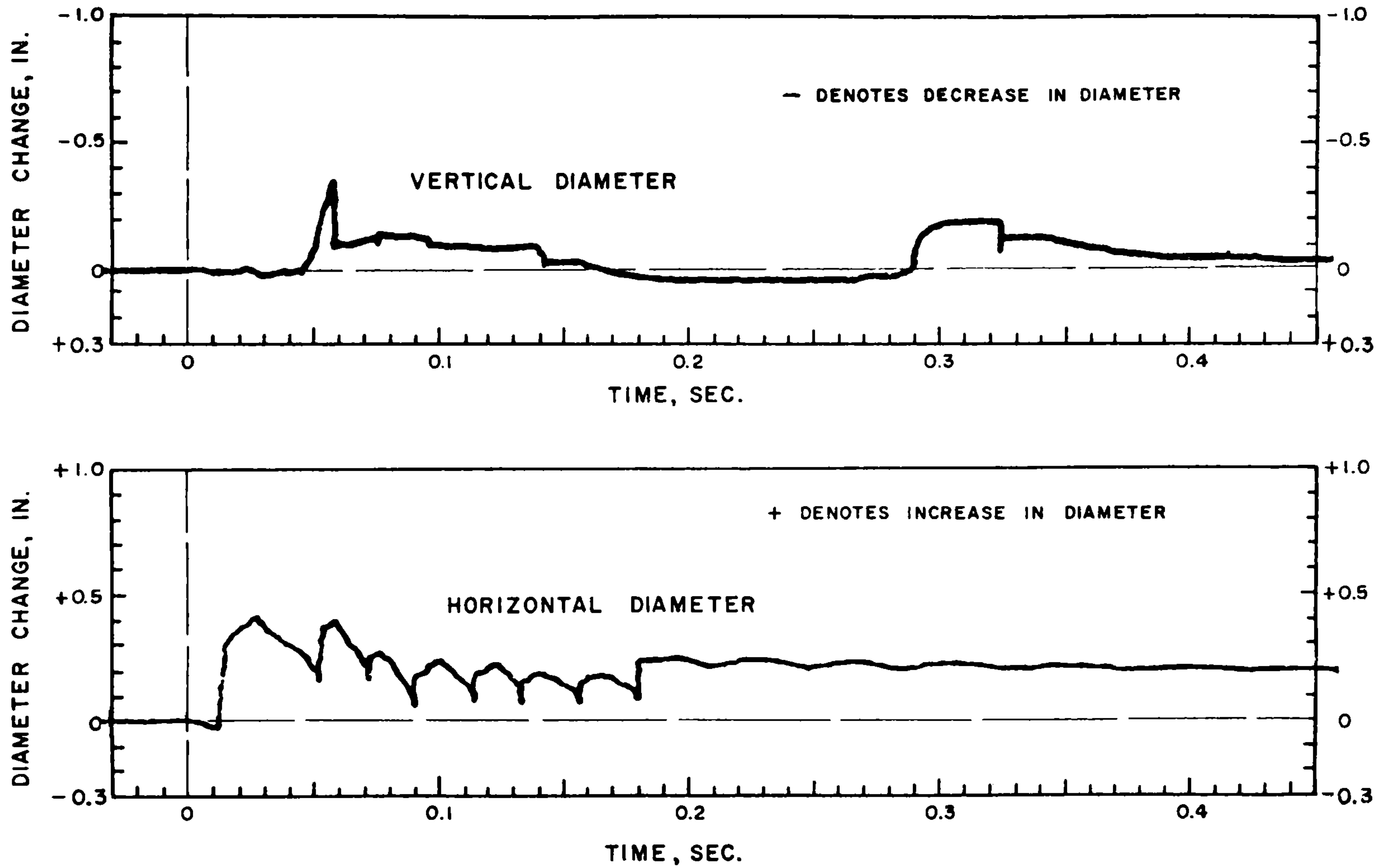


Fig. 3.3—Transient diameter changes, Station 2.

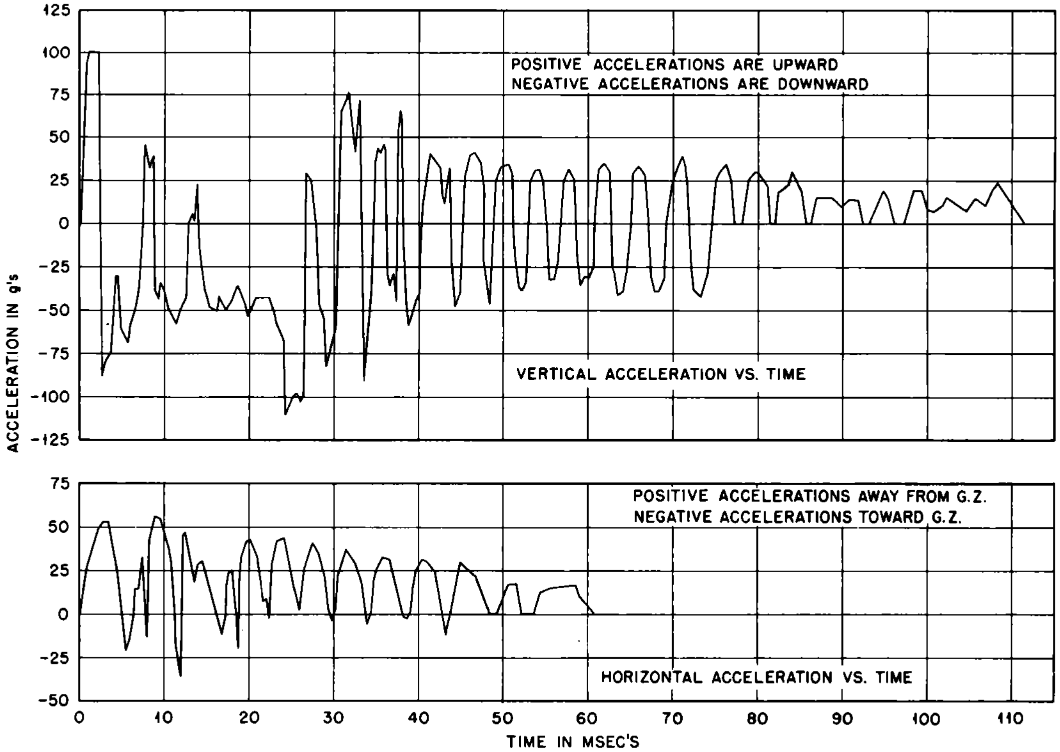


Fig. 3.4—Invert accelerations, Station 1.

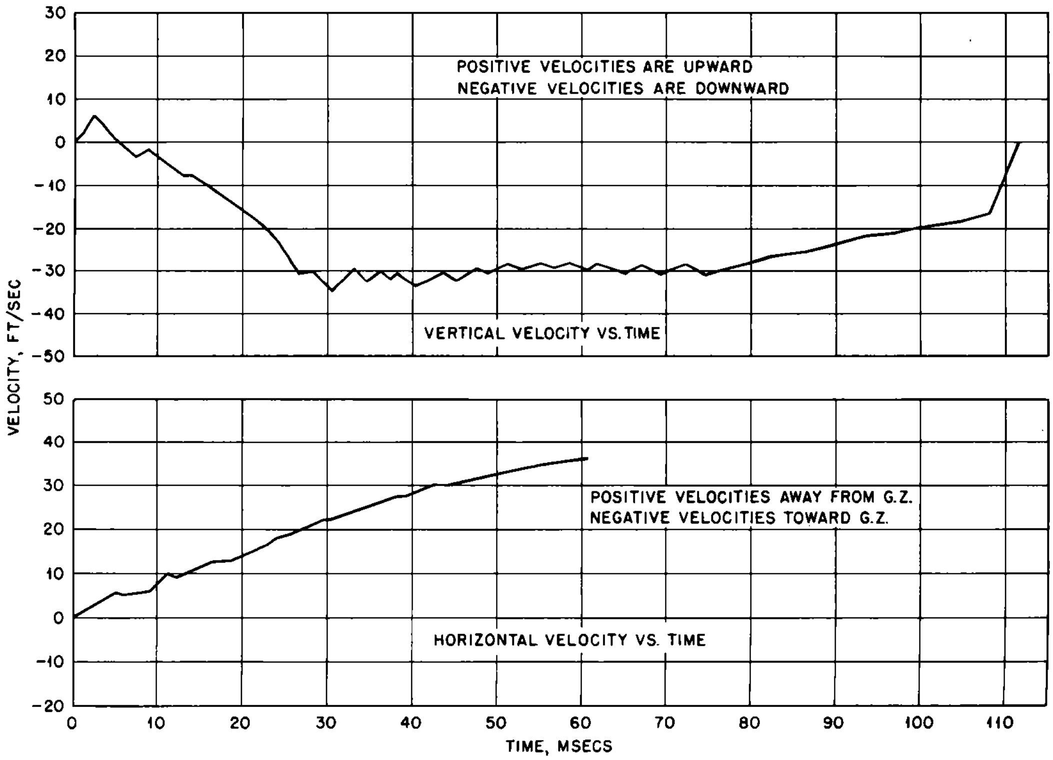


Fig. 3.5—Invert velocity, Station 1.

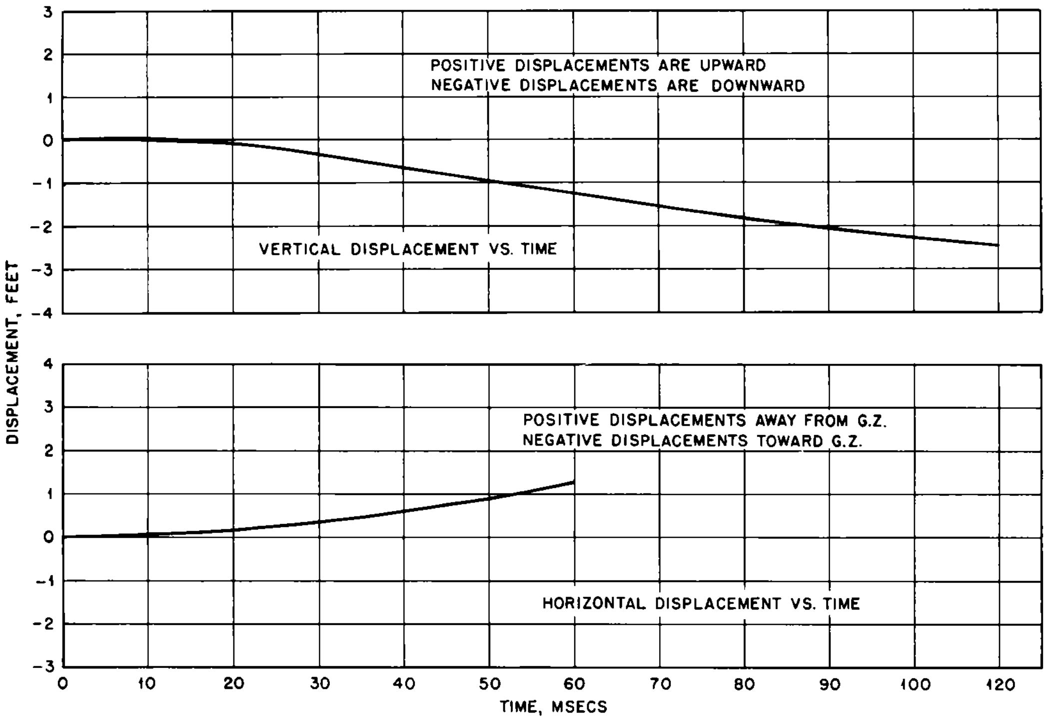


Fig. 3.6 — Invert displacements, Station 1.

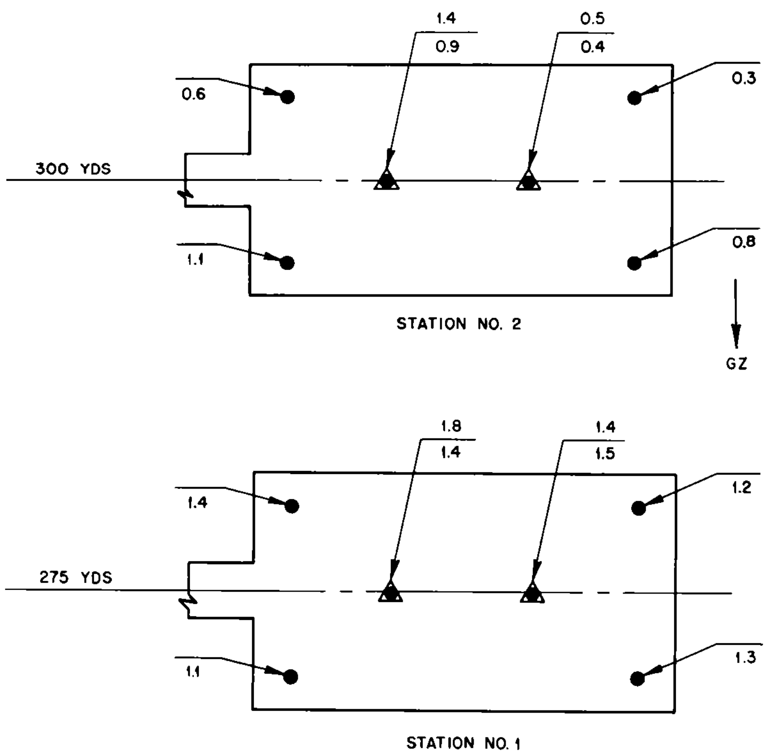


Fig. 3.7 — Film-badge locations and dosages. ●, EG&G film badges at 3-ft height. △, EG&G film badges at 3- and 5-ft height. Top reading indicates dose at 5-ft height; lower reading indicates dose at 3 ft. Doses are indicated in roentgens.

Chapter 4

DISCUSSION

4.1 OVERPRESSURES

The extrapolated values of the peak surface overpressures, 245 and 190 psi, were somewhat less than the anticipated values of 265 and 195 psi.

The maximum interior overpressure was less than 0.5 psi, which indicated that the sealing methods used around the bulkheads and the entrance pit were effective. The internal pressures were not of sufficient magnitude to affect the structural behavior of the pipe, and it is not probable that instruments would have suffered any injurious effects.

4.2 TRANSIENT CHANGES IN PIPE DIAMETERS

Transient changes in diameters were much smaller than anticipated. However, some of the characteristics of the records are of sufficient interest to justify discussion.

As shown in Figs. 3.2 and 3.3, the transient change in vertical diameter attains its maximum in a short rise time (about 0.01 sec). A secondary peak is reached between 0.2 and 0.3 sec later. Recovery from this secondary peak is small; thus the residual change in diameter is a large proportion of the secondary peak value. The record between peaks is fairly smooth, sloping downward to a minimum at the start of the second peak.

The characteristics of the transient changes in horizontal diameters are considerably different from those of the vertical diameters. The initial outward motion of the horizontal diameters appears to be preceded by a slight inward movement. The first part of the outward motion is extremely rapid; but the rate of increase suddenly diminishes near the peak, resulting in a rise time of 0.01 to 0.02 sec. The remainder of the motion consists of a series of relatively high frequency oscillations (about 40 cycles/sec) of decreasing amplitude, which finally results in a residual change that is a large percentage of the peak value. Since the frequency of the measured displacements is more than two times the natural frequency of the gauges, the readings should be reasonably free of inaccuracies caused by gauge resonance.

The horizontal diameter record also indicated a secondary peak occurring near the end of the motion; this peak was masked to some extent by the oscillations following the major peak. The motion at Station 1 lasted appreciably longer than that at Station 2. The maximum transient change in horizontal diameter (0.41 in.) was less than 0.5 per cent of the pipe diameter.

The duration of strong motion was slightly longer at Station 1 (greater than 0.30 sec) than a Station 2 (less than 0.30 sec). The peak transient change in vertical diameter was small (0.89 in., or about 1 per cent of the pipe diameter).

At Station 1 there was close agreement between time of occurrence of maximum diameter changes and time of maximum surface overpressure. This agreement was not so apparent at Station 2, where the horizontal diameter change was maximum some 30 msec prior to the overpressure peak.

No apparent relation existed between surface overpressure and the secondary peaks. A possibility that the secondary peaks were caused by the delayed arrival of a reflected seismic shock was considered. If bedrock is 700 to 800 ft* below the surface and the seismic velocity was 5000 ft/sec,* a shock reflected from bedrock would have reached the pipe approximately 0.3 sec after the direct seismic shock. This coincides with the time of the second peak on the record. In addition, the reflected shock should have caused a decrease of the vertical pipe diameter, which actually occurred.

The motion of the pipe may be explained qualitatively as a flattening of the ring section caused by the downward movement of the earth mass over the pipe and the simultaneous outward movement of the sides of the pipe. Following the initial flattening of the pipe and displacement of the earth at the sides, the sides of the pipe vibrated in the compressive mode and the horizontal diameter oscillated as indicated by the record. At the same time, the mass of earth over the top of the pipe either prevented oscillation completely or else increased the period of actual oscillation to the extent that the oscillations did not appear except, possibly, as the second peak previously mentioned.

4.3 RESIDUAL CHANGES IN DIAMETERS FROM PRESHOT AND POSTSHOT MEASUREMENTS

The residual changes in vertical and horizontal diameters given in Table 3.1 do not agree with those obtained from the deflection gauges, as shown by the comparison in Table 3.6. However, they do indicate that the residual changes are small.

Table 3.3 shows that the pattern of residual deflection is unsymmetrical about the vertical plane of symmetry of the pipe. This dissymmetry results from a shortening of one slant diameter and a lengthening of the other.

Such dissymmetry is often attributed to the fact that the part of a structure nearest GZ receives load slightly before the remainder of the structure. In the case of the pipes used in this project, this unbalance, although too momentary to be structurally serious, should cause a decrease in curvature in the upper quadrant nearest GZ. Because of this, diameter A should lengthen, and diameter B should shorten. Table 3.3 indicates that such a change occurred only at Station 2; at Station 1 the change was reversed.

4.4 INVERT ACCELERATIONS, VELOCITIES, AND DISPLACEMENTS

The vertical acceleration record of Fig. 3.4 indicates that accelerations were predominately downward for the first 30 msec. The initial motion, however, was recorded as a sharp upward spike with a peak of 100 g. The peak downward acceleration of 110 g occurred approximately 25 msec after the zero time of the record. From 40 msec to about 75 msec, upward and downward accelerations were approximately equal; and the average magnitude of the peaks in this interval was about 35 g at a frequency of 250 cycles/sec. From 75 msec to the end of the record, the accelerations were generally upward and were of decreasing magnitude.

The record of horizontal accelerations shows motion away from GZ dominating the record. The peak value was 56 g and occurred approximately 9 msec after zero time of the record.

For comparison, the recorded vertical accelerations were checked against (1) an empirical expression of Anderson;[2] (2) acceleration computed as force divided by mass, where a force equal to the peak surface overpressure was divided by the mass of pipe plus cover; and (3) records from an accelerometer buried 10-ft deep with a peak surface overpressure[3] of 205 psi. These accelerations were 39 g, 40 g (see Appendix A, Sec. A.4), and 20 g, respectively. Two buried Multiplate pipes tested in Operation Plumbbob[4] under $7\frac{1}{2}$ ft of cover experienced only 8 g under a peak surface overpressure of 153 psi.

*Reasonable values, see Ref. 1.

Both the upward and downward accelerations recorded in the first 35 msec in the present test were much larger than any of the values mentioned in the preceding paragraph, but the accelerations recorded in the time interval from 40 msec to about 75 msec are in general agreement with those values. The 40-msec time coincides with the time of occurrence of the peak overpressure (see Fig. 3.1), and a change in the appearance of the accelerometer record at this time might have resulted from the change in surface overpressure at the same time. However, it should be noted that except for the Multiplate pipes,[4] the values cited in the preceding paragraph are for free-field accelerations. Generally, higher accelerations would be expected on the perimeter of any opening such as the pipe shell than at a corresponding location in the free field.

Postoperation evaluation of the accelerometer by BRL indicated that it was inadequately damped. The elements could resonate, resulting in large outputs. This fact possibly explains why the early vertical accelerations were of such extreme magnitude.

A comparison of the acceleration-time record with values given by Dill[5] indicates that the limit of human endurance was probably exceeded during the large upward acceleration that occurred at the beginning of the record. Otherwise the recorded upward and transverse accelerations were not of sufficient magnitude and/or duration to cause injury.

Curves showing the variation of transient vertical and horizontal velocities with time are shown in Fig. 3.5. The velocities were obtained by the integration of the acceleration-time record and were subject to cumulative error if the accelerometer records were not accurate. The maximum downward velocity obtained in this manner was about 35 ft/sec, and it occurred 30 msec after record zero. The velocity then remained fairly constant for approximately 45 msec, subsequent to the maximum. The horizontal velocity appeared to increase rather uniformly to the end of the record, where its value was 36 ft/sec away from GZ.

Displacement-time curves obtained by integration from the velocity-time record are shown in Fig. 3.6. The maximum transient downward deflection obtained in this manner was about 2.5 ft; whereas the maximum horizontal deflection, which also occurred at the end of the record, was slightly greater than 1 ft.

Comparison of the curve of transient diameter change vs. time to that of derived invert displacement vs. time indicates that the diameter changes were independent of the transient absolute displacements of the pipe invert. Free-field measurements of soil motion were not obtained, but presumably the motions of the pipe and the surrounding soil were similar.

It is emphasized that velocities and particularly displacements obtained by integrating acceleration-time records reflect and amplify any inaccuracies in that record and caution should be exercised in arriving at conclusions based on the derived information.

4.5 SURFACE ELEVATIONS

The depression of the fill surface, as measured by the extent of the movement of the monuments, was about twice as great as that of the adjacent undisturbed soil. The monuments located in undisturbed soil sustained a residual decrease in elevation of from 0.09 to 0.12 ft at both stations. However, no other points in undisturbed material were surveyed; so it is not known if permanent depression of undisturbed soil was prevalent in the vicinity. The results agree with those of previous tests[3] in which a general surface depression of up to 2 in. was observed under similar conditions.

The surface depression was greatest, as expected, over the axis of the pipe; but it was only slightly greater than the depressions measured at the edges of the trench. This indicated that the friction forces, assumed to be acting between the trench walls and the backfill, were mobilized.

Residual depression of the monument over the pipe axis at Station 1 was 0.27 ft; whereas residual decrease of the vertical pipe diameter was only 0.015 ft. The absence of experimental information concerning invert elevation changes prevents a determination of the actual amount of compression of the fill over the pipe.

4.6 JOINT SLIP

No slip was evident in the longitudinal seams, which indicated that the hoop compression in the plane of the pipe cross section was not large enough to overcome the clamping action exerted by the initial tension in the bolts. The compressive stress in the pipe wall at which slip would occur is difficult to estimate, but it possibly lies in the 10 to 20 kip/sq in. range. Axial forces parallel to the longitudinal axis of the pipe were not large enough to cause slip in the circumferential seams.

4.7 DISTANCES BETWEEN END BULKHEADS

The distances between end bulkheads, measured before and after the event, changed very little, indicating that the measures taken to prevent transmission of horizontal load from the bulkhead to the pipe were adequate and/or the net horizontal load on the bulkhead due to the blast was small. Distances were measured between the faces of the neoprene seal to obtain the nearest approximation to the actual length of the pipe. In general, very little indentation of the neoprene by the pipe edge was evident.

The changes in length given in Table 3.4 for Station 2 indicate an elongation of the pipe instead of a shortening; this appears to be unrealistic. In addition, the great change in length indicated at the top of the pipe is highly questionable. It is considered desirable to discount entirely the values for Station 2 given in Table 3.4.

4.8 INTERIOR RADIATION LEVELS

The film-badge readings shown in Fig. 3.7 indicate that the gamma-radiation dose was greatly attenuated by the earth cover over the pipes. No surface radiation measurements were obtained, but the estimated dose of initial gamma radiation on the surface in the area was approximately 235,000 r. The average doses recorded inside Stations 1 and 2 were 1.4 r and 0.7 r, respectively.

Readings throughout Station 1 were fairly uniform. At Station 2, however, the following observations were made: (1) Readings near the station entrance were higher than those farther away from the entrance. (2) Readings taken at a height of 5 ft from the invert elevation were larger than those taken at a height of 3 ft. (3) Doses at points closest to GZ were larger than those at points toward the leeward side of the pipe.

As previously noted, the average dose inside Station 1 was twice as great as that inside Station 2, although Station 1 was only 20 yards (slant range) closer to the point of burst than was Station 2. No proportionate difference existed in the estimated surface dosages over the two stations, and the line-of-sight dirt thickness at Station 2 was only 1 ft greater than at Station 1 (18.3 and 17.2 ft, respectively).

The maximum total radiation recorded by any film badge was 1.8 r in the $1\frac{1}{2}$-month period between the zero time and the time of badge recovery. Under the test conditions these stations would have provided adequate protection to instrumentation and personnel against gamma radiation.

4.9 INFLUENCE OF SOIL PROPERTIES

Apparently the pipe resisted only a small fraction of the peak overpressure that acted on the ground surface. This conclusion is well supported by the small transient deflections recorded and the complete lack of slip in the bolted joints.

The primary resisting element was evidently the surrounding soil. Contributing to the resistance of the soil were the extreme density of the native material surrounding the excavation, the high degree of compaction of the backfill material, and the large ratio of depth of burial to pipe diameter.

Quantitative data regarding the soil properties of the native material were not obtained. However, the fact that the sides of the excavation stood nearly vertical and the extreme difficulty of hand excavation provide ample evidence of the large load capacity of the material.

The procedure used to determine the degree of compaction of backfill material was the modified American Association of State Highway Officials (AASHO) test method. Values obtained by its use are less than those obtained by the unmodified AASHO test method originally contemplated. Because of this the 93 per cent compaction reported probably would have corresponded to about 98 per cent on the basis of the unmodified AASHO test method. This additional compaction caused a more than proportionate increase in the structural properties of the soil.

Evidence of the extremely high passive soil modulus of the backfill material and the native material at the sides of the pipe was indicated by the small changes in pipe diameters resulting from the backfilling operations.

The 10-ft depth of backfill over the crown of the pipe was chosen to approximate the depth contemplated in the design of reinforced-concrete tunnels. It had a significant effect in reducing both the load on the pipe and the radiation level in the interior of the pipe.

The manner in which the soil carries its part of the load is obscure. One hypothesis, which has been used to explain the bridging effect of earth cover, is beam action. According to this theory, the earth above the structure functions as a beam under axial compression from the horizontal thrust due to the surface overpressure. The strength of the beam in shear is a function of the coefficient of internal friction and the horizontal thrust. Its flexural strength is limited by the requirement that the tensile stress due to flexure cannot exceed the axial compressive stress due to the horizontal thrust.

Recently published guides for the design of buried arches and domes[6] allow some attenuation of pressure with depth. The attenuation allowed is a function of the ratio of the depth of burial to the span of structure and of the shearing strength of the soil. The pipe under consideration, if designed according to the referenced procedures, would have been designed for a radial compressive load of 0.42 times the surface overpressure. If these guides and the assumptions stated in Appendix A, Sec. A.5, had been used, failure of the longitudinal seams of the pipe might have been expected if peak surface overpressure had exceeded 400 psi.

Uncertainty regarding the proper values to assign to the variables involved is one factor limiting the quantitative value of any theory involving unknown soil properties.

4.10 RELIABILITY OF DATA AND INSTRUMENTATION

Correlation between the records of the self-recording gauges and the preshot and postshot diameter measurements was poor. The preshot diameter measurements were considered to be very reliable since they were repeated at various stages of the backfilling operation with consistent results. The postshot measurement of the vertical diameter of Station 1 was verified and found to be accurate. Hence it appears that, at least in this particular case, the gauge record may be inaccurate.

On the other hand, the residual diameter changes based on the preshot and postshot measurements were not entirely consistent. According to these data, the maximum residual change occurred in the horizontal diameter for Station 2. The gauge readings show that the maximum residual change occurred in the vertical diameter of Station 1, which seems to be more realistic.

The acceleration records were of questionable quality. A postoperation evaluation of the accelerometer by BRL indicated that records obtained by this instrument should be used only to indicate gross trends.

Anomalies resulting from the preshot and postshot measurements of slant diameters and distances between bulkheads have been discussed in Secs. 4.3 and 4.7.

4.11 MISCELLANEOUS CONSIDERATIONS

The sloping entrance pipe, which terminated in an access pit at its upper end, was used for reasons of economy on the assumption that quick entry after the event would not be re-

quired. Unfortunately, high radiation levels persisted long after the event and greatly impeded the recovery of records owing to the time required to remove the sandbags from the entrance pit. The use of a much shallower pit to minimize the number of sandbags and a lightweight quick-opening cover, although more costly, would have expedited recovery.

REFERENCES

1. M. S. Johnson and D. E. Hibbard, "A Geological Survey of the Atomic Energy Commission Area, Nye Country, Nevada," Project 1.8, Armed Forces Special Weapons Project, Operation Tumbler-Snapper, WT-518, March 1953.
2. F. E. Anderson, Jr., Blast Phenomena from a Nuclear Blast, Proc. Am. Soc. Civil Engrs., 84: 1-8 (November 1958).
3. J. W. Wistor and W. R. Perret, Ground Motion Studies at High Incident Overpressure, Project 1.5, Sandia Corporation, Operation Plumbbob Report, WT-1405, October 1957.
4. G. H. Albright, Evaluation of Buried Conduits as Personnel Shelters, Project 3.2, Bureau of Yards and Docks, Operation Plumbbob Report, WT-1421, November 1957.
5. A. F. Dill, Civil Engineer Corps, USN, Effect of Blast Acceleration Forces on Personnel Within Structures, Bureau of Yards and Docks, Washington, D. C.
6. J. L. Merritt and N. M. Newmark, Design of Underground Structures to Resist Nuclear Blast, Structural Research Series No. 149, Volume II, Final Report, University of Illinois and Office of Chief of Engineers, April 1958.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The following are the principal conclusions:

1. Under conditions similar to those of this test, Multiplate pipe could be substituted for reinforced-concrete sections.
2. The portion of the blast load carried by the pipe was less than the total load acting on the ground surface. The major portion of the load was apparently carried by the fill above the pipe.
3. Friction forces between fill and trench walls were fully mobilized, as indicated by subsidence of surface monuments.
4. The small deformations of the pipe and the lack of slip in the bolted joints indicate that the peak load on the pipe was a small percentage of the load required to produce failure. Under soil conditions and depth of burial comparable to those that existed in these tests, a much higher overpressure of comparable duration would be required to cause pipe failure. This overpressure would probably be not less than that required to cause large general permanent subsidence of the ground surface.
5. In general, transient and residual pipe-diameter deformations were not proportional to the peak overpressure at the ground surface.
6. Residual deformations were a large percentage of the transient deformations, although the latter were small. Because of this it is possible that repeated blast loadings would cause further residual deformations. Such a tendency to creep under repeated loads would be an important factor when deformations are large.
7. The peak transient horizontal force acting on the bulkheads was apparently a small fraction of the peak overpressure at the ground surface.
8. A primary and a secondary peak were apparent in the gauge records of the changes in vertical and horizontal diameters vs. time. The records for the horizontal diameters indicated high-frequency oscillations not evident in the records for the vertical diameters.
9. The buried pipe, under the conditions of this test, provided adequate protection to contents against nuclear radiation. The average accumulated doses in the $1\frac{1}{2}$-month period between zero time and time of recovery were 1.4 r and 0.7 r at Stations 1 and 2, respectively.

5.2 RECOMMENDATIONS

These recommendations assume that future use of tunnels of the type described would be sufficiently extensive to justify additional tests.

1. Advance planning should lead design to the extent that the suitability of various sites can be evaluated, especially from the standpoint of subsurface soil characteristics.

2. Further tests should aim at determining the overpressure required to cause failure. The criterion for failure should be based on minimum requirements for access, which would mean that extremely large residual deformations would be acceptable.

3. Pressure-sensing instrumentation of the exterior of the pipe and/or of the soil immediately adjacent to the pipe should be considered. Results obtained from this instrumentation should give valuable information concerning the loads that actually act on the pipe as compared to surface overpressures.

4. The effect of varying depth of earth cover at a given overpressure should be determined by testing several pipes at the same range with different depths of burial.

5. The pipes should be located as close to GZ as practicable, although surface overpressure instrumentation may not be reliable at extremely close ranges.

6. Ideally the GZ site chosen should be used for several events. This would afford an opportunity to observe the creep characteristics of the pipe under repeated loads.

7. Consideration should be given to above-surface mounding rather than complete burial with the axis of the pipe oriented toward GZ. This would probably be a more severe condition in relation to blast effects than complete burial, and the loading would be more difficult to predict. However, above-surface mounding would have several advantages: (a) the surface location of some of the tunnels previously used at NTS would be simulated; (b) the results obtained could be applied conservatively to buried pipe; (c) to a great extent the influence of the native soil surrounding the excavation would be eliminated. (this is a variable that cannot be easily controlled at a given site); and (d) a substantial cost savings where excavation is difficult could result.

8. When rapid recovery of records is necessary, future tests should incorporate the means for that recovery. The higher initial cost to provide this feature may be easily offset by the alleviation of postshot access problems.

Appendix A

ANALYSIS

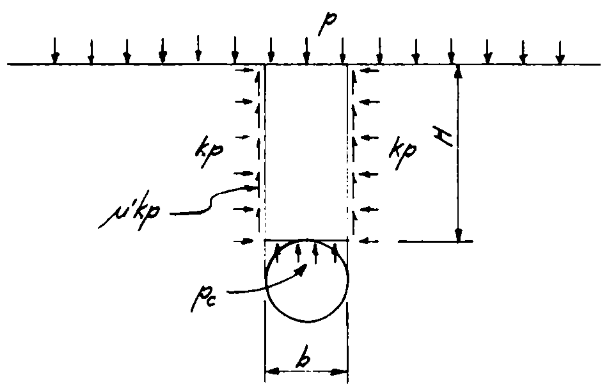
A.1 EFFECTIVE DYNAMIC LOAD ON PIPE

The purpose of this calculation is to obtain an approximate value of the percentage of the surface overpressure which acts as a load on the pipe.

Assumptions:

1. The load on the surface is transmitted to the pipe through a prism of earth directly above the pipe, and the load is reduced by friction forces on the vertical planes bounding the prism.
2. The load that acts on the pipe is applied radially.

Let p = surface overpressure
p_c = radial pressure acting on pipe
μ' = coefficient of internal friction (dynamic)
k = ratio of lateral soil pressure (due to blast) to surface overpressure
H = depth of burial
b = diameter of pipe



From the sketch:

$$p_c = \frac{1}{b}(pb - 2kp\mu'H)$$

$$= p - 2kp\mu'\left(\frac{H}{b}\right)$$

$$= p\left[1 - 2k\mu'\left(\frac{H}{b}\right)\right]$$

Assuming $k = 0.10$; $\mu' = 0.7$; and with $H = 10'$ and $b = 7'$,

$$\left[1 - 2k\mu'\left(\frac{H}{b}\right)\right] = 1 - 2(0.10)(0.7)\left(\frac{10}{7}\right)$$

$$= 1 - 0.2 = 0.8$$

Therefore $p_c = 0.8p$

A.2 RESISTANCE OF PIPE

The purpose of this section is to determine the resistance of the pipe to various modes of failure, all in terms of radial external pressure on the pipe

A.2.1 Yield Resistance

Assumptions:

1. Failure is caused by the yielding of pipe material in compression
2. The load applied is a uniform radial pressure
3. Static yield stress = 30 kips/sq in.
4. Dynamic increase = 0.2

Then f_y = dynamic yield stress = $1.2 \times 30 = 36$ kip/sq in.

$$r_1 = \frac{f_y A}{R}$$

where r_1 = resistance of the pipe as controlled by the yielding of the pipe material in pounds per square inch radial pressure

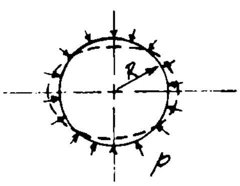
A = longitudinal cross-sectional area of the pipe skin per inch of pipe length in square inches

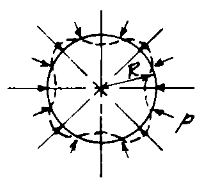
R = mean radius of the pipe in inches

A.2.2 Buckling Resistance

Two possible forms of buckling of a circular ring under uniform radial external pressure are shown below, with expressions for their critical buckling loads. The primary mode (elliptical) was not considered likely to occur because of the development of passive pressure at the sides of the pipe. The higher mode could possibly occur as a result of a localized failure of the soil.[1]

$$p_{cr} = \frac{3EI}{R^3}$$





$$p_{cr} = \frac{15EI}{R^3}$$

Resistance

$$r_2 = \frac{15EI}{R^3}$$

where E = modulus of elasticity of the pipe material = 30×10^6 psi

I = moment of inertia of the pipe skin per inch of pipe length, in.^{4}

R = mean radius of the pipe in inches

r_2 = resistance of the pipe in pounds per square inch radial pressure, as controlled by buckling

A.2.3 Resistance of Joints

(a) *Longitudinal Seams*

Let n = number of bolts per foot

P_b = ultimate resistance per bolt in shear (see Ref. 2, p. 50, Figure 41), in pounds per square inch

r_3 = pipe resistance as controlled by longitudinal bolted seams, pounds per square inch radial pressure.

Assume a dynamic stress increase = 0.2

$$r_3 = \frac{1.2nP_b}{12R} = \frac{nP_b}{10R}$$

(b) *Circumferential Seams.* The load tending to cause failure of these seams is that acting on the ends of the pipe.

Let S = bolt spacing around perimeter of the pipe
N = total number of bolts
r_4 = resistance as controlled by strength of circumferential seams

Then

$$N = \frac{2\pi r}{S}$$

and

$$kr_4 \times \pi R^2 = \frac{2\pi R}{S} \times P_b$$

$$r_4 = \frac{2P_b}{kRS}$$

A.3 SURFACE OVERPRESSURE REQUIRED TO CAUSE PIPE FAILURE

A.3.1 Material Failure

$$0.8p = r_1 = \frac{f_y A}{R}$$

For 10-gauge pipe, $A = 0.167$ sq in.

$$p = \frac{f_y A}{0.8R} = \frac{36(0.167)}{0.8(43)} = 0.175 \text{ kip/sq in.}$$

A.3.2 Buckling Failure

$$r_2 = \frac{15EI}{R^3}$$

For 10-gauge pipe, $I = 0.078 \text{ in.}^4$

$$r_2 = \frac{15(30 \times 10^6)(0.078)}{(43)^3} = 462 \text{ psi}$$

and

$$p = \frac{0.462}{0.8} = 0.578 \text{ kip/sq in.}$$

A.3.3 Longitudinal Seam Failure

Eight bolts per foot were used, so $n = 8$. However, Ref. 2 states that the failure of a joint with six or eight bolts per foot occurs in the pipe material rather than in the bolts; thus the strength of a joint with eight bolts per foot is not much greater than a joint with six bolts per foot.

$$0.8p = r_3 = \frac{nP_b}{10R} = \frac{nP_b}{430}$$

$$p = \frac{nP_b}{344}$$

For six bolts per foot in 10-gauge pipe, Fig. 41 of Ref. 2 indicates an ultimate strength of about 14.5 × 4 = 58 kips

$$p = \frac{nP_b}{344} = \frac{58}{344} = 0.168 \text{ kip/sq in.}$$

A.3.4 Circumferential Seam Failure

$$0.8p = r_4 = \frac{2P_b}{kRS}$$

$$P_b = \frac{1}{6} \times \text{strength of joint with six bolts per foot}$$

$$= \frac{1}{6} \times 58 = 9.7 \text{ kips}$$

$$p = \frac{2P_b}{0.8kRS} = \frac{2.5(9.7)}{0.10 \times 43 \times 9.6} = 0.583 \text{ kip/sq in.}$$

According to this analysis, failure of the longitudinal seams of the pipe or possibly of the pipe material itself might have been expected when surface overpressure exceeded about 0.170 kip/sq in.

A.4 COMPUTED ACCELERATIONS

(1) Anderson[3] presents the following empirical expressions for acceleration at a 10-ft depth:

$$\text{Vertical acceleration} = 0.053\, p_{so}^{1.2}$$
$$= 0.053\,(245)^{1.2} = 39 \text{ g}$$

$$\text{Horizontal acceleration} = 0.7 \times \text{vertical acceleration}$$
$$= 0.7 \times 39 = 27 \text{ g}$$

(2) $$\text{Acceleration} = \frac{F}{M}$$

where F is the force of the peak surface overpressure and M is the mass of the pipe and the earth cover. Since the plan dimensions of the pipe are 7 by 20 ft,

$$F = p_{so} \times 144 \times 7 \times 20 = 245 \times 144 \times 7 \times 20 = 4.94 \times 10^6 \text{ lb}$$

The pipe weighs 190 lb per linear foot; therefore

$$\text{Mass of pipe } M_p = \frac{190 \times 20}{32.2} = 118 \text{ slugs}$$

The soil weighs 114 lb/cu ft (see Appendix B, Table B.1), and the effective depth of the soil prism over the pipe is 7 ft (see Ref. 4, p. 34).

$$\text{Mass of soil prism } M_s = \frac{7 \times 20 \times 7 \times 114}{32.2} = 3740 \text{ slugs}$$

$$M = M_p + M_s = 118 + 3740 = 3858 \text{ slugs}$$

$$\text{Acceleration} = \frac{F}{M} = \frac{4.94 \times 10^6}{3858 \times 32.2} = 40 \text{ g}$$

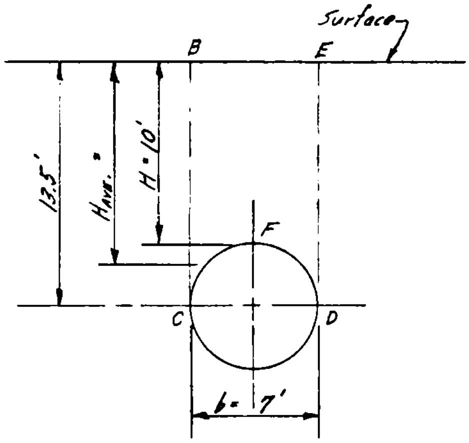
A.5 COMPARISON OF RESULTS WITH CURRENT DESIGN CRITERIA

The preliminary analysis of Sec. A.1 indicated that the pressure expected to act on the pipe would be about 0.8 times the surface overpressure. However, the pressure that actually acted on the pipe was apparently a smaller fraction than the 0.8 × surface overpressure. In light of these results, the following computation was made to compare the criteria of Ref. 4 with the results of this test.

$$H_{av.} = \frac{\text{Area BCDE} - \text{area CFD}}{b}$$

$$= \frac{13.5 \times 7 - \dfrac{\pi(3.5)^2}{2}}{7}$$

$$= \frac{94.5 - 19.2}{7} = \frac{75.3}{7} = 10.8 \text{ ft}$$



10.8 > 0.25(b) > 1.75; so the pipe is completely buried and only compressive-mode loading needs to be considered.

10.8 > 0.5(b) > 3.5; so the compressive-mode loading may be reduced below the value of surface overpressure, as follows:

$$p_c = p_{so} - \frac{H_{av.} - 0.5b}{0.5b} (c + 0.25 \, p_{so} \tan \phi)$$

where p_c = radial compressive load acting on the pipe in pounds per square inch
p_{so} = maximum surface overpressure in pounds per square inch = 245 psi
c = cohesive strength of the soil = 0 for this soil
ϕ = angle of internal friction = 48°, average (see Appendix B)

$$p_c = 245 - \frac{10.8 - 0.5(7)}{0.5(7)} [(0.25) \, 245 \tan 48°]$$

$$= 245 - 2.09 \, (68) = 245 - 142 = 103 \text{ psi}$$

Therefore

$$p_c = \frac{103}{245} = 0.42 \, p_{so}$$

rather than 0.8 p_{so}.

The period of vibration of a steel ring vibrating in the compressive mode T = (R/2600) (Ref. 5, Chapter 11), where R = mean radius in feet, is:

$$T = \frac{3.58}{2600} = 0.00138 \text{ sec}$$

Since only a compressive radial load is assumed, only the period of vibration in the compressive mode will be considered. The period T computed above must be corrected for the mass of the soil cover, but a soil mass of 7-ft depth will be used rather than $H_{av.}$ = 10.8 ft (see Ref. 4, p. 34):

$$T' = T\sqrt{\frac{M'}{M}} = 0.00138\sqrt{\frac{3858}{118}} = 0.0079 \text{ sec}$$

The rise time of the surface overpressure ≅ 35 msec (see Fig. 3.1).

$$\frac{\text{Rise time}}{T'} = \frac{35}{7.9} = 4.4 > 2$$

so consider p_c to be a static load on the pipe (see Ref. 4, p. 41).

If a value of p_c equal to 168 psi would cause failure of longitudinal joints, a surface overpressure in excess of (168/0.42) = 400 psi would cause failure of these joints.

REFERENCES

1. G. H. Albright, Evaluation of Buried Conduits as Personnel Shelters, Project 3.2 Bureau of Yards and Docks, Operation Plumbbob Report, WT-1421, November 1957.
2. Handbook of Drainage and Construction Products, Armco Drainage & Metal Products, Inc., Middletown, Ohio, 1955.
3. F. E. Anderson, Jr., Blast Phenomena from a Nuclear Blast, Proc. Am. Soc. Civil Engrs., 84: 1-8 (November 1958).
4. J. L. Merritt and N. M. Newmark, Design of Underground Structures to Resist Nuclear Blast, Structural Research Series No. 149, Volume II, Final Report, University of Illinois and Office of Chief of Engineers, April 1958.
5. Holmes & Narver, Inc., Design Manual, AEC Test Structures, in preparation.

Appendix B

SOIL PROPERTIES

B.1 FIELD DENSITY AND COMPACTION TESTS

Field density tests were taken according to the AASHO Standard Test Method T-147 modified to use a 6-in.-diameter sand cone having a free fall of 6 in. Bag samples of representative fill materials were obtained for compaction tests.

Compaction tests were performed on representative soils according to the AASHO Standard Test Method T-99 modified to use 25 blows of a 10-lb hammer falling 18 in. on each of three layers of soil in a 4-in.-diameter cylindrical mold of $\frac{1}{30}$ cu ft volume. Maximum density of the backfill material was 122 lb/cu ft, attained at a moisture content of 10.5 per cent. Results of the density and the compaction tests are given in Table B.1.

TABLE B.1 — PRESHOT FIELD DENSITY TESTS

Test No.	Location*	Date	Field moisture, %	Field density, lb/cu ft	Max. density, lb/cu ft	Per cent of maximum
1	North pipe	July 27, 1957	11.3	111	122	91
2	North pipe	July 27, 1957	9.8	110	122	90
3	North pipe	July 27, 1957	9.3	105	122	86†
4	South pipe	July 28, 1957	14.1	112	122	92
5	South pipe	July 30, 1957	14.3	113	122	93
6	South pipe	July 30, 1957	11.4	115	122	94
7	South pipe	July 31, 1957	14.9	116	122	95
8	South pipe	Aug. 1, 1957	10.4	118	122	97
9	North pipe	Aug. 2, 1957	12.9	110	122	90
10	North pipe	Aug. 2, 1957	10.8	113	122	93

*North pipe is Station 1; south pipe is Station 2.
†Loose material was removed and recompacted.

B.2 GRADATION

Table B.2 gives the results of the sieve analysis.

TABLE B.2 — GRADATION

Sieve size	Per cent retained	Per cent passing
No. 4	0.3	99.7
No. 10	39.3	60.7
No. 20	56.9	43.1
No. 40	70.1	29.9
No. 60	79.3	20.7
No. 100	84.3	15.7
No. 200	87.8	12.2

B.3 COEFFICIENT OF INTERNAL FRICTION

Direct shear tests were made on saturated specimens of the typical materials used for compacted fills; these materials were remolded to 90 per cent of maximum density, with a direct shear machine of the constant strain type at a rate of strain of 0.05 in./min. Table B.3 gives these results.

TABLE B.3 — ANGLE OF INTERNAL FRICTION

Station	Distance above bottom of pipe, ft	Coefficient of friction	Angle of internal friction
1	7	1.42	55°
2	6	0.875	41°